THE RELATIONSHIP BETWEEN HIGHWAY GEOMETRICS, TRAFFIC RELATED ELEMENTS, AND MOTOR VEHICLE ACCIDENTS

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Poisson and negative binomial regression techniques were used as a means to predict accidents on the basis of highway geometrics and traffic related factors. For a particular highway section the overall number of accidents was predicted using both Poisson and negative binomial distributions. The predictions were then compared with actual accident statistics. Both methods use a log-linear function to ensure that accident prediction is always non-negative. The primary data sources were the Washington State Department of Transportation's Transportation Information and Planning Support database for geometric and traffic information and the Washington State Patrol's accident database for accident information. The maximum likelihood method was used to estimate the model coefficients. The results suggested that horizontal curvature, daily traffic, speed, number of lanes, and tangent length between curves are significantly correlated with accident occurrence. The results indicated that if accident data are dispersed relative to the mean, negative binomial regression is the most appropriate method of analysis.

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DISCLAIMER

TABLE OF CONTENTS

LIST OF TABLES

CHAPTER 1
INTRODUCTION

Motor vehicle accidents continue to be a leading cause of death and injury in the United States. In 1992 the National Safety Council (NSC) estimated the capital cost of each accident to be $880,000 for a death, $29,500 for a non-fatal disabling injury accident, and $6,500 for property damage only (National Safety Council, 1992). The willingness to pay approach produces substantially higher costs, such as approximately $3,200,000 for a fatality accident (Federal Highway Administration (FHWA), 1991; National Safety Council, 1992; Mannering and Winston, 1993).

HIGHWAY SAFETY INVESTMENT PROGRAMMING IN WASHINGTON STATE

Understanding the substantial economic and social burden of motor vehicle accidents on our society, the Washington State Department of Transportation (WSDOT) continuously searches for ways to reduce the number of accidents on state highways. As a means of reducing the frequency and severity of accidents, WSDOT proposed a proactive programming method to the Washington State Legislature. Subsequently, the legislature passed into law the Priority and Programming Act of 1993 (Revised Code of Washington 47.05). This legislation required WSDOT to develop a safety funding program to address both high accident and "high risk" locations on the state highway system. Simply defined, high risk locations have a high probability of accident occurrence.

In the past, WSDOT had programmed highway safety investments through the High Accident Location (HAL) program and as part of paving (preservation) projects. WSDOT had applied "safety standards" in the pavement preservation program to get federal matching money for preservation projects. As a result of this fiscal reality, WSDOT had programmed safety dollars because of the need to preserve pavement rather than because of the need to apply safety standards.

In accordance with RCW 47.05, WSDOT's methods of priority programming for safety have changed. The safety improvement priorities are now separated from pavement preservation priorities. Whereas about 12 percent of safety restoration will be applied to the paving program, the majority of safety money will be programmed in the safety improvements subprogram. This means that the routine application of standards will be based on safety need rather than on preservation need. Driving the changes in RCW 47.05 was the belief of WSDOT and the Washington State Legislature that the routine application of safety standards to preservation projects is not the most cost effective way to program safety dollars. Rather, it is most cost effective to program safety projects where the safety needs exist.

By law, safety projects must now be based on both historic accident data and the risk (probability) of accidents at a location. To identify locations of need, WSDOT divided its safety subprogram into two categories. One is the Accident Prevention Category. The other is the Accident Reduction Program Category. The Accident Reduction Program includes the High Accident Locations (HAL) and High Accident Corridors (HAC) subcategories. The HAL subcategory is intended to identify a location on a highway that has a high incidence of accidents (above the statewide average for similar highways) over the previous two-year period. The HAC is similar to the HAL, but it uses a half-mile rolling section to identify any mile-long section of highway that demonstrates a high incidence of accidents over three two-year periods. In addition, three criteria must be met for a mile to be considered a HAC; two of these are measures of severity and one is an accident frequency measure.

Both of these methods of accident identification are reactive. Often WSDOT must wait for a series of collisions to occur before a section of highway becomes a high enough priority to program. The ability to improve known hazardous locations is further impaired because accident frequency varies significantly from year to year. A section of highway that is a high priority one year may not be as high the next. By using a statistical model that determines the probability of accidents occurring on the state highway system, WSDOT will be better able to program safety projects proactively and avoid waiting for a series of accidents to occur.

In 1993, WSDOT began to develop the Accident Prevention Category of the Safety subprogram. The Accident Prevention Category will address areas of high risk. A subcategory of Accident Prevention will focus on areas that have the highest probability of accident occurrence based on highway geometrics for non-interstate highways. Initially, the Risk subcategory will use a modification of the methods described in the *American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide* (Appendix A). However, WSDOT considers these procedures to be interim measures until a more definitive method, as defined by this project, is developed.

CHAPTER 2
LITERATURE REVIEW

MODELS

A number of researchers have shown that the cause of a vehicle accident is a complex set of events involving the interactions of many factors. A number of statistical models have attempted to determine the specific characteristics that are most significant in determining vehicle accident probability and accident rates for specified highway sections (Roy Jorgensen Associates, Inc., 1978; Zegeer, Hummer, Reinfurt, Herf, and Hunter, 1987; Okamoto and Koshi, 1989; Zegeer, Reinfurt, Neuman, Stewart, Council, 1991; Miaou and Lum, 1993; Miaou et al., 1992; Miaou, 1994; Shankar, Mannering and Barfield, 1995; Poch and Mannering, 1995). In addition, the Federal Highway Administration (FHWA) published the report *Safety Effectiveness of Highway Design Features*. This compilation of research focuses on studies between 1973 and 1991. Most of the studies contained in the report used conventional linear regression techniques to model the interaction between highway geometrics and accidents. However, linear regression is reported to have undesirable statistical properties in describing vehicle accidents. A number of studies have proposed possible solutions to this and other statistical problems (Jovanis and Chang, 1986; Joshua and Garber, 1990).

Jovanis and Chang found a number of problems with the use of linear regression in their study. The authors discovered that as vehicle miles traveled (VMT) increases, so does the variance of the accident frequency. This is a direct violation of the homoscedasticity assumption of linear regression. The effect of this violation invalidates the hypothesis test because the confidence intervals are affected. This makes it difficult to validate the significance of the estimated parameters. As the authors concluded, if the objective of a study is to determine the influence that particular predictor variables have on accident occurrence, the failure to properly test for parameter significance is a serious flaw.

The second problem is that linear regression is not restrained from predicting negative accident occurrence. This would be a significant factor where a main road section has a low or no accident frequency for some period of time. Negative accident prediction will bias the estimated coefficients, invalidating the model unless corrective measures are taken.

To account for these known violations of linear regression, Jovanis and Chang suggested and used Poisson regression as a means to predict accidents. They argued that the statistical properties of Poisson regression are superior to those of linear regression for applications regarding highway safety.

In their 1990 study, the authors modeled the occurrence of accidents using both linear regression and Poisson regression with accident, travel mileage, and environmental data from the Indiana Toll Road. The pooled models revealed that accident occurrence increases with automobile vehicle miles of travel (VMT), truck VMT, and hours of snowfall.

Similarly, Joshua and Garber (1990) studied the relationship between highway geometrics and truck accidents in Virginia . Both linear and Poisson regression models were developed. They also concluded that the multiple linear regression techniques used in their research did not describe the relationship between truck accidents and the independent variables adequately but that the Poisson models did.

The finding that linear regression did not adequately describe the relationships of vehicle accidents and highway characteristics led to the use of Poisson and negative binomial (NB) regression in more recent research (Miaou, Hu, Wright, Rathi and Davis, 1992; and Miaou and Lum, 1993; Miaou, 1994; Shankar, Mannering and Barfield, 1995; Poch and Mannering , 1995).

Miaou et al. (1992) used a Poisson regression model to establish the empirical relationship between truck accidents and highway geometrics on a rural interstate in North Carolina. The estimated Poisson model suggested that AADT per lane, horizontal curvature, and vertical grade were significantly correlated with truck accident rates, but shoulder width was much less correlated. The authors also found overdispersion in the Poisson model, which was most likely due to uncertainties in the data or omitted variables.

The authors stated that although overdispersion existed, it did not change the conclusion about the relationship between truck accidents and the examined traffic and highway geometric design variables. The authors also concluded that the Poisson model should be applied to other types of roadways. Where overdispersion or underdispersion exist, the authors suggest exploring other discrete distributions such as negative binomial distribution.

A follow-up study was completed by Miaou and Lum (1993). While this study was similar in scope to the first, the main purpose was to evaluate the statistical properties of two conventional linear regression models and two Poisson regression models. The models studied by Miaou and Lum were comparable to those used in previous studies to develop the relationship between vehicle accidents and highway geometric design. The models were examined in terms of their distributional assumptions, estimation procedures, function form, and sensitivity to short road sections. The study also discussed issues pertaining to these models such as the treatment of vehicle exposure, traffic conditions, and potential data uncertainties due to sampling and non-sampling errors.

The four types of models were (1) an additive linear regression model, (2) a multiplicative linear regression model, (3) a multiplicative Poisson regression with exponential rate function and, (4) a multiplicative Poisson regression with nonexponential rate function.

The authors found in a comparison of the regression models that the Poisson regression models outperformed linear regression models. Furthermore, the Poisson regression model with the exponential rate function was the favored model.

Miaou and Lum concluded that the conventional linear regression models lack the distributional property to describe random, discrete, non-negative, and typically sporadic vehicle accidents on the road. They also found that these models are sensitive to short road sections. More importantly, the authors stated that there was no assurance that the expected total number of accidents predicted would be close to the observed totals. Therefore, the use of these linear regression models was inappropriate for making probabilistic statements about the occurrences of vehicle accidents on the road.

The authors found that while the Poisson regression models possessed most of the statistical properties needed to describe vehicle accidents, they too were not without problems. In the Poisson regression model the variance of the data is constrained to be equal to the mean. The research showed that because of this the variances of the estimated model coefficients tended to be underestimated when overdispersion existed in the data in a Poisson model. Miaou and Lum attempted to relax this constraint by using Wedderburn's overdispersion parameter. They found that with such overdispersed data, using the Poisson model may not be appropriate for making probabilistic statements about vehicle accidents because the model may under- or overestimate the likelihood of occurrence. Because of the overdispersion difficulties, the authors suggested the use of a more general probability distribution, such as the negative binomial.

In 1994, Miaou studied the relationship between highway geometrics and accidents using negative binomial regression (Miaou, 1994). In this study Miaou evaluated the performance of the Poisson regression, zero-inflated Poisson (ZIP) regression, and negative binomial regression. The maximum likelihood was used to estimate the unknown parameters of the models. In addition, a moment-based estimator and a regression-based estimator were used to evaluate the negative binomial.

When using the maximum likelihood, Miaou found that the models were quite consistent and that no model outperformed the other. However, he warned that when the negative binomial regression is used with either the moment or regression based estimator, researchers should be cautious.

As an initial step in developing a model, the author suggested that the Poisson regression model be used to establish the relationship between highway geometrics and accidents. If overdispersion exists and is found to be moderate or high, both the NB and ZIP regression models can be explored. Overall, Miaou suggested, the ZIP regression model appears to be more appropriate when the data exhibit excess zeros.

In 1995 Shankar, Mannering, and Barfield used both the Poisson and negative binomial distributions to evaluate the effects of roadway geometrics and environmental factors on rural accident frequency in Washington State. In addition to modeling the total accident frequency on sections of highway, they modeled the associated severity for different accident types. The authors reasoned that separate regression models would have greater explanatory power when describing accidents.

The accident frequency models were classified into sideswipe, rear-end, parked vehicle, fixed object, overturn, and same direction accidents. The estimation results revealed that the negative binomial model was most appropriate for all accident types, with the exception of overturned vehicles.

Poch and Mannering (1995) developed a model to identify the most significant traffic and geometric elements in predicting accident frequency. This study used both the Poisson and negative binomial regression models during the analysis. Four different accident models were developed. The models included total accident, rear end, angle, and approach-turn accident frequencies. The authors found left turn volume and total opposing approach volume to be highly significant in prediction and that, in all cases, the negative binomial was the appropriate model for determining accident frequency.

Previous research has shown that linear regression is not the most appropriate means for evaluating the statistical relationship between vehicle accidents and highway geometrics. Instead, Poisson regression and negative binomial regression are more statistically suitable. However, the vast research that has used linear regression techniques still gives significant empirical insight into the development of models that use other statistical methods. For this reason the following section discusses the empirical relationships between accidents, geometrics, and roadway characteristics.

ACCESS CONTROL

Access control is established to preserve the safety and efficiency of specific highways and to preserve the public investments (WSDOT 1989). In both urban and rural areas, access control will range from fully controlled to uncontrolled access on different classes of highway. However, most highways are constructed with limited or no access control. The exception is the interstate system, which remains, for the most part, fully access controlled.

The Effect of Access Control and Safety

AASHTO states that the most significant design factor related to safety is access (AASHTO, 1990). The amount of access is directly related to the number of unexpected events and conflicts. Therefore, attempts are made to eliminate as many of these decision points as possible. The number of access points can be reduced by implementing regulatory access controls, grouping access points to a single point, spacing access points, and many other techniques. By using these techniques, engineers and planners are able to improve traffic operation and reduce accident potential.

The benefits of these techniques were quantified in a Congressional Report that used data from 30 states. The report concluded that full control of access is the most important design factor related to crash reduction (Stover, Tignor and Rosenbaum, 1982). This report showed that the average accident rates on rural highways were about one half as high for highways where access was fully controlled as rates on rural highways where access was uncontrolled. For urban highways, accident rates were about one third as high on highways with fully controlled access as on those without access control.

A 1959 report by the Bureau of Public Roads compared the safety record of existing primary highways with the newly constructed interstate system to determine the safety of the interstate. The results of this report were documented in a number of later follow-up studies. The most comprehensive, by Fee, Beatty, Dietz, Stephen and Yates, showed results similar to the earlier congressional study. The Fee et al. study indicated that accident rates are nearly three times higher on urban highways with partially controlled access than accident rates on urban interstate with full access control. The study also indicated that accident rates on rural highways with no access control are twice as high as rates on similar rural locations with partial access control (Fee, Beatty, Dietz, Stephen and Yates, 1970). The results of these models consistently showed access to be a significant variable in the reduction of accidents.

Median Openings

In a number of studies on the effects of median openings on accidents, Cribbins, Arey and Donaldson (1968) found that volume and various measures of access were significantly correlated with traffic accidents. The authors indicated that under low volume conditions, wide medians, and little development, median openings are not necessarily dangerous. However, as volume and development increase, the use of median openings increases accident occurrence. This research also found that median openings are involved in about 35 percent of accidents that occur between intersections on four-lane divided highways. A conclusion of the research was that the number of median openings should be kept to a minimum (Cribbins, Arey and Donaldson, 1968). The authors found that in all cases, accident rates increased with an increase in the number of access points and signalized openings per mile. The authors further concluded that when storage lanes were present, median widths were not a significant factor in crash reduction on multi-lane facilities.

A significant point of this study was that a combination of geometric and traffic characteristics was found to have a more significant impact on accidents than single variables; therefore the authors advised against conducting further studies involving the use of single variables.

Roadside Access

Driveways provide access to and exit from many highway facilities. Unlike intersections, driveways often do not receive attention to ensure that they are properly controlled or designed. In many cases, the necessity for access to and exit from a facility takes precedence over the need for access control.

A common assumption is that the safety of driveways is a function of average daily traffic (ADT). With this belief, significant research has been undertaken to quantify the effects of ADT and access on accidents rates. Two such studies found that as ADT and commercial access increase, so do accident rates. (Fee, Beatty, Dietz, Kaufman and Yates, 1970; McGuirk, 1973). The McGuirk study also found that the number of lanes, intersections, driveways per mile, and urban area population have an effect on accident rates.

A more thorough study by Glennon and Azzeh (1976) identified 77 countermeasures on existing non-controlled access facilities and evaluated many of them by using conflicts instead of accidents as the measure of effectiveness. Glennon and Azzeh's countermeasures focused on the reduction of access points, such as closing median openings, frontage road access, and driveways with access to both frontage roads and main facilities. The authors found that separating the through vehicles from turning vehicles reduced conflicts and accidents. They recommended the extensive use of turning lanes, both acceleration and deceleration, for driveway and non-signalized intersections. To provide a better understanding of the impacts driveways have on accident rates, the authors developed Table 1. The table shows estimates for the number of driveway accidents by access points and ADT.

TABLE 1. ANNUAL NUMBER OF DRIVEWAY ACCIDENTS PER MILE BY FREQUENCY OF ACCESS AND TRAFFIC VOLUMES

Driveways per Mile	Highway ADT		
	Low <5,000	Medium 5-15,000	High >15,000
<30	12.6	25.1	37.9
30-60	20.2	39.7	59.8
<60	27.7	54.4	81.7

ALIGNMENT AND CURVE CHARACTERISTICS

Horizontal and vertical alignment are critical elements in highway safety. Logically, one can expect that accidents will occur more often on curves or grades than on relatively straight tangent sections. In sections where drivers must make more decisions the likelihood of accidents increases because of a greater likelihood of driver error. The result is that engineers prefer to minimize the effect of alignment by designing the highway with as gentle a curve or grade as feasible.

The vertical or horizontal alignment of a highway is designed from the concept of safe stopping sight distance. Stopping sight distance is the length of roadway ahead visible to the driver. The minimum distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path (AASHTO, 1990). Designing the highway by using the concept of stopping sight distance reduces the total number of decision points, and therefore, reduces accidents.

Vertical and horizontal alignments should be designed with consideration for each other. Poor design of one can affect the positive aspects or deficiencies of the other.

Vertical Alignment

Two types of vertical curves are crest and sag. Vertical curves are typically designed to accommodate application of standards and ease of construction, and to produce a safe and comfortable driving experience. In general, three major factors affect the design of vertical alignment: safety, terrain, and construction costs. This section discusses the interrelationship between safety and vertical alignment.

A study by Brinkman and Perchonk indicated that the accident rates for downgrades are about 63 percent higher than those for upgrades. Table 2 shows that injury and fatality rates on vertical curves are higher on downgrade sections than on upgrade locations (Brinkman and Perchonk, 1979).

TABLE 2. ACCIDENT FREQUENCY AND SEVERITY BY VERTICAL ALIGNMENT

Vertical Alignment	Number of Accidents	Percent of Total Accidents	Percent Injured	Percent Killed
Level	2001	34.6	53.6	4.7
Upgrade	943	16.3	55.6	3.9
Downgrade	1533	26.5	58.4	5.1
Up on Crest	373	6.5	59.5	6.0
Down on Crest	461	8.0	62.6	5.9
Up on Sag	258	4.5	57.8	6.3
Down on Sag	211	3.7	61.7	6.8
Total	5780	100.0		

A similar study found that 69 percent of the through-lane accidents on the Central Expressway in Dallas occurred on crest and sag vertical curves. In an attempt to directly relate the effect of sight distance, the authors related accident frequency to the algebraic difference in grades, sight distance value, K, and the highway volume. They hypothesized that the accident frequency would be directly proportional to the algebraic difference and volume, and inversely proportional to the sight distance, which is dependent on the length of vertical curve for any value of algebraic differences. Although they found no direct correlation in terms of accidents and the curves, the authors still concluded that a general lack of sight distance contributes to high accident rates at the peaks of crests and the uphill portions of sag vertical curves (Mullins and Keese, 1961).

In a Transportation Research Board report to Congress, Glennon (1987) reviewed stopping sight distance. He found that alignment changes undertaken to increase very short sight distances to very long distances appear to be safety effective. However, Glennon stated that these sight distance improvements may only be cost effective when traffic volumes are very high and major hazards are hidden by sight obstructions.

Horizontal Alignment

A number of studies have shown that accident rates are higher at horizontal curves than at tangents. Horizontal curve accident rates range from one and a half to four times greater than those at tangent sections (Smith, Purdy and McGee, Harwood, St. John, and Glennon, 1983; Glennon, Newman and Leisch, 1985; Zegeer, Stewert, Council and Reinfurt, 1991). Significant research has also quantified the characteristics that most affect the safety performance of horizontal curves. These studies have concluded that the most important characteristics of horizontal curves include the following:

- traffic volume and mix on the curve
- curve features (degree of curve, length of curve, central angle, superelevation, and the presence of spiral or other transition curves)
- cross-sectional curve elements (lane width, shoulder width, shoulder type, and shoulder slope)
- roadside hazards on the curve (clear zone, side slope, rigidity, and type of obstacles)
- stopping sight distance on curve
- vertical alignment on a horizontal curve
- distance to adjacent curves
- presence/distance from curve to the nearest intersection, driveway, bridge
- pavement friction

- presence and type of traffic control device.

Zegeer et al. (1991) studied the effects of various curve-related improvements on accidents. The data consisted of 10,900 horizontal curves in Washington State. The study found that the types of accidents that occur more frequently on horizontal curves than on tangents include head-on, opposite direction, and sideswipe crashes; fixed-object and rollover crashes; crashes at night; and those involving drinking drivers. The study also found that the length of curve, ADT, degree of curve, width of roadway, and presence of spiral were significant.

Glennon, Newman, and Leisch (1985) also developed a model for using geometrics, traffic, and roadside conditions to identify horizontal curve sites that have a high potential for accidents. The authors found that curves with sharper curvature, greater length, more hazardous roadsides, lower skid resistance, and narrower shoulders have a higher probability of accident occurrence.

A number of studies have shown that sharper curves are more susceptible to accidents than broader curves (Glennon, et al., 1985 and 1987; Deacon, 1986; Zegeer et al., 1991). The Zegeer study showed a reduction in accidents for varying degrees of curve flattening improvements. The study also described percentage reductions in curve related accidents as a result of shoulder width or lane width increases. However, the authors warned that percentage improvements are not additive when accident reduction factors are combined (Zegeer, Reinfurt, Neuman, Stewart and Council, 1991). Improving sharp curves is often a costly undertaking, and the authors suggested that such improvements only be undertaken when abnormally high accident rates exist.

Spiral Transitions

Glennon et al. (1985), used field studies of path behavior on unspiraled curves to find that drivers, while attempting to spiral their path from an infinite radius to the radius of the highway curve, always overshoot the curve radius. This overshooting creates higher frictional demands. The authors reported that adding spiral transition to highway curves dramatically reduces the frictional demands of the critical vehicle traversals (Glennon, et al., 1985).

This conclusions was supported by the Zegeer et al. (1991) study, which discussed the safety effectiveness for spiral transitions on high speed horizontal alignments. The model revealed that, depending on the degree of curve and central angle, spiral transitions reduced curve crashes by 2 to 9 percent (Zegeer, Reinfurt, Neuman, Stewart and Council, 1991).

Superelevation Improvements

The superelevation of a curve is another factor that is critical to the curve's proper functioning. A number of studies have attempted to link superelevation to accident causation (Zador, Stein, Hall and Wright, 1985; Zegeer, Reinfurt, Neuman, Stewart and Council, 1991). The study by Zador et al. found that the available superelevation at fatal accident sites was typically deficient. In the Zegeer et al. study, the authors noted that curves with too little superelevation had worse accident rates than curves with proper superelevation. The study concluded that curves with superelevation below the AASHTO guideline, as specified in *A Policy on Geometric Design of Highway and Streets* have significantly worse accident rates than those with superelevation above the minimum guideline.

Zegeer et al., used Table 3 to quantify the effects of superelevation improvements in their 1991 FHWA report.

TABLE 3. ACCIDENT REDUCTION FACTORS CORRESPONDING TO IMPROVING SUPERELEVATION

e_d	Accident Reduction Factor Due to Upgrading Superelevation
001 to 0.019	5 percent
≥0.02	10 percent

CROSS SECTIONS

The cross section of the highway is of major importance to roadway safety. The characteristics of the cross section are often the determining factors in whether the driver and vehicle will crash or safely recover from an errant maneuver. Cross sectional elements such as lane width, shoulder width, and shoulder type have been found to be very important (Jorgenson 1978; Zegeer et al., 1987b).

No feature of a highway has a greater influence on the safety and comfort of driving than the width and condition of the driving surface (AASHTO, 1990). In a 1987 Federal Highway Administration (FHWA) report by Zegeer, Hummer, Reinfurt, Herf and Hunter, nearly 5,000 miles of two-lane highway were studied to quantify the effects of lane width, shoulder width, and shoulder type on highway accidents (Zegeer et al., 1987b). The authors concluded that lane and shoulder conditions directly influenced run-off-road and opposite direction fixed object, rollover, head on and sideswipe accidents. Other accident types, such as rear-end and angle accidents, were not directly affected by these elements. Accident rates were also shown to drop as lane width increases. To quantify the expected effects of lane and shoulder widening on rural roads, an accident prediction model was developed. The Zegeer et al. study produced a number of tables that can be used to estimate percentage of reduction in accidents for differing improvements. Table 4 shows the percentage of reduction in accidents due to lane widening, paved shoulder widening, and unpaved shoulder widening.

The effects of roadway widening on rural, farm-to-market roads was quantified in a 1989 study by Griffin and Mak (Griffen and Mak 1989). The study showed that as road widths increase for varying degrees of ADT, single-vehicle accident rates decrease.

Numerous other studies from various states have quantified the effects of lane and shoulder width changes, as shown in Table 5.

TABLE 4. PERCENTAGE OF REDUCTION IN ACCIDENTS DUE TO LANE WIDENING, PAVED SHOULDER WIDENING, AND UNPAVED SHOULDER WIDENING

Total Lane or Shoulder Widening (ft.)		Percentage of Accident Reduction		
Total	Per Side	Lane Widening	Paved Shoulder Widening	Unpaved Shoulder Widening
2	1	5	4	3
4	2	12	8	7
6	3	17	12	10
8	4	21	15	13
10	5	--	19	16
12	6	--	21	18
14	7	--	25	21
16	8	--	28	24
18	9	--	31	26
20	10	--	33	29

TABLE 5. COMPILATION OF PREVIOUS RESEARCH REGARDING LANE AND SHOULDER WIDTHS

		Factors Considered		
Author	Date	Shoulder Width	Shoulder Type	Lane Width
Dart and Mann	1970	X	X	
Heimbach, Hunter, and Chao	1974			X
Foody, Long	1974	X	X	X
Shannon and Stanley	1976	X	X	
Rinde	1977		X	
Jorgensen and Associates	1978	X	X	X
Sager, Mayes, Deen	1979	X	X	
Turner, Fambro, Rogness	1981			X
Rogness, Fambro Turner	1982			X
Zegeer and Deacon	1987	X	X	X
Zegeer, Hummer Reinfurt, Herf, Hunter	1987	X	X	X
Griffin and Mak	1989			X

CHAPTER 3
RESEARCH APPROACH

The key issue of this research was the development of statistical models for use in predicting traffic vehicle accidents. The models were developed using the correlation between traffic accidents and highway characteristics (including geometrics and traffic mix). The relationship between predicted and actual accidents was quantified using Poisson and negative binomial regression methods.

In developing accident prediction models, one must understand that vehicle accidents are the result of a complex set of interactions involving many variables. In many, if not all, cases of highway accident prediction, one cannot determine, nor control, all the factors that may have contributed to a particular accident's occurrence.

For instance, the models developed in this research do not account for all human factors that may contribute to an accident. This could be a fundamental flaw of accident prediction models. However, as Massie, Campbell and Blower (1993) suggested, using a human factors approach to classifying accidents and their related causes fails to address the issue of helping drivers avoid a collision. The authors argued that by identifying geometric conditions that lend themselves to producing accidents, the problem conditions can be corrected (Massie, Campbell and Blower, 1993). Furthermore, because it is difficult, if not impossible, for the highway designer to control for every action that a driver might make, correcting locations with questionable or deficient geometrics reduces the locations where accidents are most likely to occur.

VARIABLE DEFINITION

This research utilized 45 highway or traffic related variables from the Washington State Department of Transportation's Transportation Information and Planning Support (TRIPS) database. From these 45 variables, an additional 16 independent variables were developed by using various combinations of the collected variables. Table 6 separates each of the variables into similar groups and briefly explains each.

TABLE 6. VARIABLES BY TYPE AND USE

Section Identifiers	
• State Route Number	Indicates state route number.
• Related Roadway Type	Each state route is accompanied by a unique identifier, including: mainline, couplet, spur, alternative route, and turn back.
• Beginning Adjusted Route Milepost	The adjusted route milepost indicates the actual miles from the beginning of the highway.
• Beginning Milepost	The milepost identifies reference points on a highway, not necessarily the actual miles from the beginning of highway.
• Length of Section	Highway section length is defined by a change to any of the collected variables.
Cross-Section Related	
• Increasing Lane	Number of lanes in the direction of increasing mileposts.
• Decreasing Lane	Number of lanes in the direction of decreasing mileposts.
• Total Lanes	Total number of lanes in a section.
• Roadway Width	The width of the roadway, excluding shoulder widths.
• Narrow lanes	Indicator variable for lane widths of less than 11.5 feet.
• Left Shoulder	The width of the left shoulder width (decreasing direction of travel).
• Left Center Shoulder	The width of the left median side shoulder (decreasing direction of travel).
• Right Center Shoulder	The width of the right median side shoulder (increasing direction of travel).
• Right Shoulder	The width of the right shoulder (increasing direction of travel).

Table 6. Continued

• Narrow Left Shoulder	Indicator variable for shoulder widths of less than 5 feet.
• Narrow Center Left Shoulder	Indicator variable for shoulder widths of less than 5 feet
• Narrow Center Right Shoulder	Indicator variable for shoulder widths of less than 5 feet
• Narrow Right Shoulder	Indicator variable for shoulder widths of less than 5 feet.
• Curb	Indicator variable for the presence of a curb on any shoulder.
• Wall	Indicator variable for the presence of a wall on any shoulder.
• Median	Indicator variable for the presence of a divided highway.
Location Variables	
• District	Identifies the WSDOT Region in which a particular section of highway is situated.
• City	Identifies the city in which a particular section of highway is situated.
• National Highway System	Identifies whether a section of highway has been designated as part of the proposed National Highway System.
• Urban/Rural	Indicator variable for urban or rural highways.
Traffic Related	
• Speed	Posted speed limit expressed in miles per hour.
• Functional Class	Designates the functional class of a highway, expressed as principal arterial, minor arterial or collector highway.
• AADT	Average annual daily traffic (AADT) per lane average.
• Med. AADT	Indicator variable for lane volumes of greater than 5000 veh. per day.

Table 6. Continued

• Single Trucks	Single unit trucks, in which the cargo area and the power unit are mounted. Expressed as a percentage of traffic mix.
• Double Trucks	Tractor plus one semitrailer—typically has three, four, or five axles. This truck is commonly known as an 18-wheeler. Expressed as a percentage of traffic mix.
• Train Trucks	Tractor plus semitrailer plus full trailer—known as a "double" or "double bottom" truck. Expressed as a percentage of traffic mix.
• Trucks	The total truck percentage for a given section of highway.
• Peak Hour Factor	A measure used to relate the peak 15 minutes with the peak hour volume.
<u>Alignment</u>	
• Vertical Curve	Indicates whether an observation contains an angle point or point of inflection.
• Vertical Grade Ahead	Indicates the vertical grade of a section.
• Cumulative Tangent Distance	A running summation of tangent distance from the previous horizontal curve.
• Tangent length 1	Indicator variable for tangent lengths that are greater that .25 miles long.
• Tangent length 2	Indicator variable for tangent lengths that are greater that .5 miles long.
<u>Horizontal Curvature</u>	
• Horizontal Curve Type	Indicates the beginning of horizontal curve, end of horizontal curve, or angle point.
• Length of Curve	The length of a horizontal curve in feet.
• Direction of Curve	Indicates whether the curvature is to the left or right in the increasing milepost direction.
• Radius	Total horizontal curve radius in feet.
• Degree of Curve	The degree of horizontal curvature.
• Central Angle	Total central angle in degrees.

Table 6. Continued

• Sharp Curve	Indicator variable for a curve that is greater than 2 degrees.
• Really Sharp Curve	Indicator variable for a curve that is greater than 5 degrees.
• Super Sharp Curve	Indicator variable for a curve that is greater than 10 degrees.
• Tangent curve 1	Indicator variable for a degree of curve greater than 2 and a tangent length ahead of a curve that is greater than .25 miles long
• Tangent curve 2	Indicator variable for a degree of curve greater than 2 and a tangent length ahead of a curve that is greater than .5 miles long
• Tangent curve 3	Indicator variable for a degree of curve greater than 10 and a tangent length ahead of a curve that is greater than .25 miles long
• Tangent curve 4	Indicator variable for an observation that has a degree of curve greater than 10 and a tangent length ahead of a curve that is greater than .5 miles long.
• Intersections	The number of intersections present in a section.
<u>Accidents</u>	
• 1992 Accidents	Total number of 1992 accidents.
• 1992 Construction Accidents	Total number of 1992 construction accidents.
• 1993 Accidents	Total number of 1993 accidents.
• 1993 Construction Accidents	Total number of 1993 construction accidents.
• Total Accidents per Year	Total accidents per year, excluding construction accidents. Used as the dependent variable for all models.

DATA COLLECTION

The most current geometric and roadway data for the Washington State highway system were accessed through WSDOT's TRIPS system. This database processes, stores, and reports information that describes the state's highway system. The information includes roadway geometrics, structures, classifications, geographic jurisdictions, and off-road features related to state highways.

The accident data were accessed through WSDOT's Accident Data Office. The Accident Data Office uses information from the Washington State Patrol's accident database for crash information.

The TRIPS roadway data system is segmented and partitioned into areas called Related Roadway Types (RRT). RRTs allow the user of the information to determine the highway type. The Related Roadway Types are mainline, collector distributor, couplet, reversible lane, high occupancy vehicle lane, alternative routes, spurs, and turn back routes. For the purpose of this investigation, the following RRTs were used: highway mainline, couplet, spurs, alternative routes, and turn back routes. Using the state route designation in combination with the related roadway type produces a unique identification of each highway. For instance, mainline SR 20 may function differently than the lesser traveled and designed SR 20 spur in Anacortes.

The RRT data were separated by physical location (i.e., Eastern and Western Washington) and highway function class. The three functional class models developed for Eastern Washington and the three for Western Washington included the following highway functional class types: principle arterials, minor arterials, and collectors.

Roadway and Geometric Data

To investigate the relationship between accidents and highway geometrics, data were collected from highways of varying characteristics and functional classes. The data included information from 7,020 centerline miles of highway and over 80,000 accident records. WSDOT supplied these highway data from its Highway Geometrics Databases. The Washington State Patrol (WSP) accident data files provided the accident history for each highway section.

The limits of a section were defined by changes to any geometric or roadway variable (e.g., a new section would be identified when the shoulder width changed from 6 feet to 8 feet). For all state highways, 87,095 highway sections were identified. The section-defining information included changes to district number, urban or rural location, state route number, related roadway type, number of lanes, roadway width, shoulder width, presence of curb or retaining wall, divided or undivided highway, speed, average annual daily traffic, truck percentage, peak hour factors, and vertical curve and horizontal curve characteristics.

These 87,095 highway sections formed eight subgroups for analysis. The subgroups were separated into Eastern and Western Washington highways, and then by interstate, principal arterials, minor arterials, and collectors. The Eastern and Western Washington split was an attempt to capture the significant weather and terrain variations that exist between the eastern and western parts of the state. An initial review revealed that the database that was produced for the models lacked a significant number of variables for the interstate (e.g., ramp information, auxiliary facilities, and lane use information was not included). Because the this project focused on non-interstate highways, and because the omission of data and variables would create bias, the researchers determined that modeling the interstate would not provide significant benefit.

For each section, shoulder widths were recorded as reported by the TRIP system. In cases where a section of highway contained curbs or walls, an indicator variable was used for identification because no shoulder width data was maintained for these sections.

Horizontal and vertical curve information was identified by the geometric data supplied from the WSDOT highway geometric database. Vertical curves were identified by grade and the presence of an angle point or inflection point. Horizontal curves were identified by the length, radius, central angle, superelevation, horizontal curve type, and the direction of curvature. The superelevation information for the highways was sporadic, and this variable was subsequently dropped from further investigation.

Accident Data

The research used a two-year summary of accident data, from January 1, 1992, to December 31, 1993, for the analysis. More than one year of data was desirable because of the variability of accident frequency from year to year. Road sections that had undergone construction during the study period were identified from construction accidents in the databases and excluded from the calculations. In addition to the highway geometric data, the computerized state roadway inventory files were used as a source of traffic volume, truck percentage, peak hour factor, geometric, and speed data.

MODEL DEVELOPMENT

Accidents can be analyzed by many means. Yet certain statistical characteristics can lead to the choice of one method over another. A review of accident data shows that the majority of data either equal zero or have small non-negative values. Because the dependent variable is clearly discrete, the researcher is led to consider the Poisson regression method of analysis in modeling accident frequency. In addition, because data show that most highway sections have a relatively higher probability of being observed with zero accidents, as opposed to more than one accident, the underlying distribution is rightly skewed. Normal approximations are not a good predictor under these situations.

However, the Poisson distribution can also be problematic. When accidents are modeled with the Poisson regression, the variance of the dependent variable Y_i (i.e., predicted accident) is often over- or underdispersed relative to the mean. Therefore, alternative methods for describing the interaction between accident frequency and geometric factors may need to be considered.

The negative binomial is one method that has shown promise for describing the interaction because the negative binomial regression relaxes the requirement that the variances of the predicted coefficients be equal to the mean. The negative binomial model is an extension of the Poisson model and includes a gamma distributed error term.

The Poisson model is defined by the following equation:

$$p(Y = y_i) = p(n_i) = \frac{\lambda^{n_i} \exp(-\lambda_i)}{n_i!} \quad i = 1,2,3\ldots,z. \qquad \text{(eq. 1)}$$

where

$$\lambda_i = E(n_i) = \exp\left(\sum_{i=1}^{z} \beta_i X_{ij}\right) \quad i = 1,2,3,\ldots,z. \qquad \text{(eq. 2)}$$

where $p(n_i)$ is the probability of (n) accidents occurring on a highway section i, and λ_i is the expected the accident frequency (i.e., $E(n_i)$) for highway section i. For each explanatory vector X_{ij}, which describes the section's geometric or traffic characteristics, there is estimated β_i, coefficient vector. To relieve the non-negativity constraint of accident prediction, the model assumes an exponential rate function, $\lambda_i = \exp\left(\sum_{i=1}^{z} \beta_i X_{ij}\right)$.

The likelihood function that results using the standard maximum likelihood methods for equations (1) and (2) and is described in detail by Lawless (Lawless, 1987):

$$L(\beta) = \frac{\prod_i \exp[-\exp(\beta X_i)][\exp(\beta X_i)]^{n_i}}{n_i!} \qquad \text{(eq. 3)}$$

However, if the variance is over- or underdispersed relative to the mean, Poisson regression is statistically troublesome because the β_i vector may be biased. The negative binomial regression is a commonly used method that relaxes the equidispersion constraint.

Defined by Greene (1992), the negative binomial is an extension of the Poisson regression model that allows the variance of the predicted coefficients to differ from the mean:

$$\ln \lambda_i = \beta X + \varepsilon \qquad \text{(eq. 4)}$$

where the error term for the expression λ_i is $\exp(\varepsilon)$. The error term has a gamma distribution with mean 1 and variance α. This gives the resulting conditional probability of

$$P(n_i|\varepsilon) = \frac{\exp[-\lambda_i \exp(\varepsilon)][\lambda_i \exp(\varepsilon)]^{n_i}}{n_i!} \qquad \text{(eq. 5)}$$

where $x = \exp(\varepsilon)$ and is gamma distributed. We can assume that f(x) is given by the following:

$$f(x) = \frac{x^{(1/\alpha)-1}}{\alpha^{1/\alpha}\Gamma(1/\alpha)}\exp\left(-x/\beta\right) \qquad \text{(eq. 6)}$$

$$x > 0 \quad \alpha > 0$$

such that $E(x_i) = 1$ and Var $(x_i) = \alpha$, then

$$P(n_i|\varepsilon) = \frac{\exp[-\lambda_i x][\lambda_i x]^{n_i}}{n_i!}\frac{x^{(1/\alpha)-1}}{\alpha^{1/\alpha}\Gamma(1/\alpha)}\exp\left(-x/\beta\right) \qquad \text{(eq. 7)}$$

Integrating over x produces the unconditional distribution of n_i. The resulting probability distribution of the negative binomial is

$$P[Y = y_i] = p(n_i) = \frac{\Gamma\left(1/\alpha + n_i\right)\ [\alpha \exp(x\beta)]^{n}}{\left[\Gamma\left(1/\alpha\right)n_i!\right][1+\alpha \exp(x_i)]^{n+1/\alpha}} \qquad \text{(eq. 8)}$$

which is the negative binomial distribution with: $E(n|x) = \exp(x_i)\beta$ and,

$$\text{var}(n|x) = \exp(x_i\beta)[1 + \alpha\exp(x_i\beta)] \qquad \text{(eq. 9)}$$

where the additional parameter, α, is referred to as the dispersion parameter.

The statistical significance (t-statistic) of the α term determines the appropriateness of the negative binomial relative to the Poisson regression. This model allows the variance to exceed the mean. However, if α is not significantly different from zero, the negative binomial reduces to the Poisson regression with Var [y_i] = E[y_i]. Table 7 shows the mean, variance, and the significance of the α variable for each of the six models.

TABLE 7. MEAN AND VARIANCE OF THE DEPENDENT VARIABLES

Accident Variable	Mean	Variance	α (t-stat)
W. Wash. Principal Arterials	0.39490	1.0496	26.335
E. Wash. Principal Arterials	0.07854	0.0354	6.529
W. Wash. Minor Arterials	0.14569	0.1431	5.519
E. Wash. Minor Arterials	0.07147	0.0730	3.697
W. Wash. Collectors	0.24402	0.0203	11.143
E. Wash. Collectors	0.10179	0.1022	5.081

CHAPTER 4
FINDINGS

The effects of geometrics and roadway characteristics have been studied using a number of statistical methods. This research focused on the Poisson and negative binomial regression methods, developing a total of six accident prediction models. Three of these were functional class models were developed for Western Washington and three for Eastern Washington. The functional classes included principal arterials, minor arterials, and collector highways.

Two specification issues were identified during the modeling process. The first was found when a cursory review of the data revealed that cross street information was not included at intersections. Because this presented a possible specification problem (omitted variable bias), highway sections containing intersections were rejected. The second issue involved changed conditions of the highway. Therefore, any section that had undergone construction during the two-year study period was identified from accient reports in the TRIPS databases and and rejected.

THE VARIABLES

The independent and indicator variables for the different highways included in the final models are discussed to provide insight into the variables' tendency to increase or decrease the frequency of accidents. The discussion below is not meant to be inclusive of every variable tested in the models. The variables that were tested and were not found to be statistically significant have been omitted. Tables 8 through 19 show the summary statistics (tables 8, 10, 12, 14, 16, 18) and model results (tables 9, 11, 13, 15, 17, 19).

The Length of a Section

Finding An increase in section length tends to increase accident probability for all functional classes of highways in Eastern and Western Washington.

Tables 9, 11, 13, 15, 17, 19

According to the length variable coefficient, as the length of a section increases, so does the accident frequency. This is consistent with intuition and logic, which suggest that short sections are less likely to experience accidents than longer sections because of decreased exposure.

<u>**Section Lengths Longer than .25 Miles**</u>

Finding An increase in section length tends to increase accident probability for principle arterials in Western Washington.

Table 9

This results is consistent with the previous finding that as the length of highway section increases, the exposure to accidents also increases.

<u>**The Vertical Grade of a Section**</u>

Finding An increase in the vertical grade tends to increase accident probability for principle arterials in Eastern and Western Washington.

Tables 9, 11

This variable is intended to assess the effects of vertical grades on collisions. In Western Washington, vertical grades greater than 1 percent were found to increase accidents. In Eastern Washington, vertical curvature was not found to influence collisions until 2.5 percent. Typically, for uphill grades, vehicles slow as the grade or the length of grade increases. This is especially true for trucks. For significantly long grades, trucks often slow to 10 mph. This reduction in speed is associated with increased passing and risk taking by faster moving cars. It seems reasonable, therefore, that as risk taking increases, accidents also increase. In contrast, downgrades have the effect of increasing vehicle speeds. This contributes to an increase in accidents because drivers have less decision time to avoid problems.

Improving the vertical alignment of a section is one method of reducing accidents. Unfortunately, improving alignment is very expensive, and therefore, not likely to be cost beneficial in most cases. Other improvements such as passing lanes may be cost effective for steep grades and should be considered.

The Average Annual Daily Traffic (AADT) per Lane of a Section

Finding An increase in AADT has a strong tendency to increase accident probability for all highways in Eastern and Western Washington.

Tables 9, 11, 13, 15, 17, 19

Because AADT is not given as directional information from the TRIP database, AADT is divided by the number of lanes in both directions of a section of highway. This gives an average volume per lane over the entire section.

The positive coefficients of the AADT in the model indicate that as the number of vehicles through a section increases, so does the number of accidents. The explanation for this is that as the number of vehicles increase through a section, the exposure to potential accidents and the number of conflicts also increase.

There is little that can be done to reduce the number of accidents at high volume sections with the exception of increasing capacity or improving the positive guidance of the section. By improving the guidance (channelization), the number of conflicts and major decision points that a driver must negotiate can be reduced.

Average Annual Daily Traffic of a Section over 2500 Vehicles

Finding An increase in AADT tends to increase accident probability for principal arterials in Western Washington.

Table 9

There is a positive correlation with accidents at locations with an AADT per lane of over 2500. An increase in the number of vehicles increases the number of conflicts and exposure to potential accidents.

Peak Hour Percentage

Finding Higher peak hour percentages tend to decrease accident probability on collectors and principal arterials in Western Washington and on minor arterials in Eastern Washington.

Tables 9, 17, 15

Intuitively, higher peak hour percentages increase accidents at many locations. For instance, stop controlled intersections or locations with recurrent stop and go conditions often experience higher accidents rates. However, these models do not include intersections, and the majority of these highways do not experience stop and go conditions. Therefore, the decrease in accident probability is most likely caused by a reduction in speeds and the lack of sudden stops associated with intersections and congestion.

Single Unit Trucks as a Percentage of Traffic Mix

Finding An increase in the percentage of trucks tends to decrease the overall accident probability for principal arterials in Western Washington and for minor arterials in Eastern Washington.

Tables 9, 15

Previous research has suggested that as the percentage of trucks increases in relationship to cars, the frequencies of vehicle overtaking and lane changing decrease (Miaou, 1994), reducing risk taking and thus accidents. The models' findings support this theory.

Caution should be used when interpreting this variable. Where AADT is low, truck percentages rapidly increase with only a few additional trucks. Hence, where AADT is low, accidents probably decrease because of a lack of conflicts, not because of increased truck traffic.

Percentage of Tractor Plus Semitrailer (18-Wheeler) Trucks in Relation to Traffic Mix

Finding An increase in the percentage of trucks tends to increase the accident probability for collectors in Western Washington and to decrease accident probability for collectors in Eastern Washington.

Tables 17, 19

The majority of collector highways in the state of Washington are two lanes with speeds of over 50 miles per hour. Therefore, passing a truck and semitrailer often requires moving into the opposing lane of traffic at high speeds. An increase in the number of head-on, sideswipe, and run-off-the-road accidents can be expected if these passing maneuvers are taken without due care or with improper judgment of the distance of an oncoming vehicle. On collector highways with relatively high AADT (as in Western Washington) the ability to overtake double trucks is minimized by a lack of gaps. This lack of available gaps, combined with impatient motorists and reduced truck speeds from steep grades, increases accident potential because under these conditions, drivers often attempt to pass without exercising due care. Where AADT is relatively low, as in Eastern Washington, drivers are able to pass with a relatively low risk of an accident because of the lack of oncoming vehicles. Once again, for collectors in Eastern Washington, the low AADT and the lack of conflicts, rather than increased truck traffic, may be the cause of a decrease in accident frequency.

A common method of reducing the potential for accidents caused by a high percentage of trucks is the addition of passing lanes on steep grades or where queuing causes increased risk taking.

Percentage of Tractor Plus Semitrailer Plus Full Trailer Trucks (Train Trucks) in Relation to Traffic Mix

Finding An increase in the percentage of trucks tends to decrease the overall accident probability for collectors in Western Washington and to increase the probability in Eastern Washington.

Tables 17, 19

On average, "train trucks" in Western Washington account for a very low percentage of the traffic mix (0.46 percent average). Given the inability to find gaps in oncoming traffic in high AADT sections and the increased length of these trucks, Western Washington drivers have very little ability to pass. Therefore, less risk taking activity occurs and accidents decrease. On the other hand, Eastern Washington drivers, who have an abundance of gaps, may experience a false sense of security with the low AADT and the low percentage of train trucks and may be willing to attempt passing without realizing the length of the truck. This leads to an increase in accident frequency as the percentage of these trucks grows in proportion to the traffic mix.

The posting of "long truck" warning signs would help to mitigate problems caused by excessive truck length.

<u>**Percentage of All Trucks in Relation to Traffic Mix**</u>

Finding An increase in the percentage of trucks tends to decrease the overall accident probability for principal arterials in Eastern Washington.

Table 11

The percentage of trucks in Eastern Washington accounts for a relatively high percentage (15 percent) of the traffic mix. Ordinarily, these facilities have more than two lanes so ample opportunity exists to pass safely without driving in the opposing lane. In addition, Miaou's (1994) assumption holds true, that as the percentage of trucks increases in relationship to cars, the frequency of vehicle overtaking and lane changing decreases, with an accompanying decrease in accidents.

<u>**Posted Speed of the Highway**</u>

Finding For all highways, with the exception of collectors and principal arterials in Western Washington, increased speed has a tendency to increase accident probability. For principal arterials in Western

Washington, as speed increases, the probability of accidents tends to decrease. For Western Washington collectors the variable was insignificant.

Tables 9, 11, 13, 15, 19

The speed variable in this model is the posted speed as opposed to the design speed. The posted speed was used because the TRIPs database does not include information regarding the design speed of a highway. However, the researchers assumed that, in general, the higher the speed limit, the higher the design speed for most highways. The researchers also assumed that, in general, the higher the design speed, the better the geometric conditions of highways. Nevertheless, design speed does not mitigate the fact accidents are likely to increase at higher speeds because of the additional time necessary for a vehicle to stop when traveling at a faster rate. At higher speeds, a vehicle travels farther between the time a driver perceives a hazard and the time he or she reacts to it. This tends to increase the number of accidents. In addition, at higher speeds driver error is accentuated, and mistakes are more costly and severe.

Principal arterials in Western Washington show an increased accident frequency where posted speed are lower. This is explained by the fact that it is necessary to lower speed limits through an urban area to reduce the severity of accidents. Many urban areas in Western Washington with reduced speed limits are located near highly congested facilities; therefore, these highways could be expected to have increased conflicts and accidents. Because of this factor, raising the speed limit cannot be assumed to reduce accident frequency. Raising the speed limit should not be used as a means to reduce accidents unless thorough engineering studies are undertaken that prove this is a reasonable and viable option.

If speed limits are to be maintained at a safe and enforceable rate, they must be set at a rate that is reasonable. Experience shows that when operating a vehicle, most people drive at a speed within their comfort zone. Simply, most drivers tend to drive at a speed they feel is reasonable regardless of the speed limit. For example, many vehicles exceed the speed limit on the interstate where design standards are for 70 mph and the speed limit is set at 55 mph.

Special care should be taken in setting speed limits. Speed limits should be set at the 85th percentile speed. In areas where speed limits must be dropped below the 85th percentile speed, special enforcement or traffic calming devices may be needed.

The Number of Lanes of a Highway Section

Finding More lanes tend to increase the number of accidents for all classes of highways.

Tables 9, 11, 13, 15, 17, 19

The variable for the number of lanes in a section is highly significant, with a positive coefficient. A number of explanations are possible for this variable's significance. As the number of lanes increase, the amount of traffic within the section is likely to increase. In addition, the number of lanes also has a significant impact on the characteristics of traffic flow. For instance, the amount of lane changing will increase with the number of lanes. The lane changing increases the number of conflicts between traffic, precipitating increased driver error and accidents.

The Lane Width of a Section

Finding Wider lanes tend to increase the number of accidents for collector highways in Eastern Washington.

Table 19

For collector highways in Eastern Washington the average lane width is 11.46 feet, and the smallest width is 9 feet. The standard lane width for a rural or urban two-lane facility is 12 feet. However, the standard lane width for multi-lane facilities is 11 feet, and the lanes are divided by a median area. One reason for the finding may be that as the width of the lane increases for a section, people can also be expected to drive faster. As stated before, this has a tendency to increase accident probability. A second plausible explanation is that the variable is picking up the effects of the median separation on the highway.

Less Than Standard Lane Widths

Finding Less than standard lane widths tend to increase the probability of accidents for minor arterials in Eastern Washington and for collectors in Western Washington and to decrease accident probability for Eastern Washington principal arterials.

Tables 11, 15, 17

Narrow is an indicator variable that describes the presence of "substandard" lanes. The model showed the variable to decrease the accident probability for principal arterials in Eastern Washington. For these roads, reduced lane width often occurs in sections where speed limits are reduced. Drivers may also sense the reduced lane width and thus reduce the their speeds. Therefore, it makes sense that the number of accidents would be reduced through these sections.

Motorists on minor arterials in Eastern Washington and collectors in Western Washington, having driven substandard highways, may not feel the need to adjust their speeds accordingly. This may result in increased accident potential.

Width of Right Shoulder

Finding As the shoulder width increases, the accident probability for minor arterials in Eastern Washington tends to increase.

Table 15

For very low volume roads, such as the collectors and minor arterials of Eastern Washington, shoulder widths have little effect on the number of accidents because the exposure to these sections is low. The Eastern Washington minor arterial model indicated a positive correlation between increasing right shoulder width and accident probability. A simple explanation for this is that drivers are lulled into a false sense of security by the increased shoulder width and tend to increase speeds and exhibit unsafe driving behavior.

Substandard Right Shoulders

Finding Substandard right shoulders tend to increase the frequency of accidents for principal arterials and collectors in Western Washington and for principal arterials in Eastern Washington.

Tables 9, 11, 17

A number of studies have quantified the effects of shoulder widths (see Table 2). The majority of those studies found that substandard shoulders have a positive correlation with the number of accident that occur on any given section of highway. This is consistent with the fact that drivers have less room to take corrective actions after making an errant maneuver. It is also apparent that drivers are more apt to encounter roadside obstacles with the reduced widths, as these are often the reason that reduced widths are necessary.

As studies have shown, shoulder widths have a direct effect on accident frequency. This indicates that shoulders should be widened as much as economically feasible.

Substandard Left Shoulders

Finding Substandard left shoulders tend to increase the accident probability for principal arterials and collectors in Western Washington and for principal arterials in Eastern Washington.

Tables 9, 11, 17

For the variable Narrow Left Shoulder, "1" indicates the presence of a shoulder of less than or equal to 5 feet in the decreasing direction of mileposts, and "0" indicates a shoulder wider than 5 feet. Similar to the situation with a narrow right shoulder, drivers have less room for corrective action and are more likely to encounter a fixed object.

Right Center Shoulders of Less Than 5 Feet

Finding Substandard right center shoulders tend to increase the accident probability for minor arterials in Western Washington.

Table 13

Like the previous variable, the variable for narrow right center shoulders attempts to determine the effects of shoulders on multi-lane highways. For minor arterials in Western Washington, the variable is significant.

On multi-lane facilities AADT is higher. Therefore, highways with median shoulders likely have higher accident rates simply because of greater exposure.

A common action for many motorists on multi-lane facilities is to abandon their vehicles on the inside shoulder if it appears that a vehicle will be reasonably safe there, rather than attempt to move their vehicles to the right shoulder. However, where the shoulder is narrower than 8 feet and wider than 4 feet, a vehicle will not be entirely off the traveled way. Therefore, designers should refrain from building inside shoulders that are between 4 and 8 feet when possible.

The Presence of a Wall within a Section of Highway

Finding The presence of a wall tends to increase the accident probability for collectors in Eastern Washington.

Table 19

This variable captures the effects of retaining and rock walls on accidents. A review of the descriptive statistics indicates that such walls are present in about 4 percent of all sections. This leads to the conclusion that drivers may be using the shoulder for driving because few wall hazards are present. Motorists, sensing that shoulder driving is safe, do not anticipate upcoming hazards and are unable to react.

Moving the wall may be one means of reducing these type of accidents; however, such relocation is not always cost effective. An alternative method is to use signs to alert drivers to possible hazards at frequent accident locations.

The Presence of a Divided Highway

Finding Divided highways tend to decrease the accident probability for collectors in Eastern Washington.

Table 19

It is intuitively obvious that a divided collector highway is bound to experience fewer accidents than an undivided highway because vehicles have more room for error and correction. This variable did not appear to be significant for other classes of highways. The explanation is that the majority of Washington State highways (excluding interstate) are undivided. For these highways, the data set would experience a particularly small amount of variance and a correspondingly low t-statistic.

The Presence of Curves Greater than 2 Degrees

Finding Curves of more than 2 degrees tend to decrease the accident probability for all classes of highways in Washington.

Tables 9, 11, 13, 15, 17, 19

Analysis indicated that curves of more than 2 degrees tend to reduce the frequency of accident occurrence. This fact seems to defy intuition. However, one factor that must be considered with this variable is the presence of nearby curves; if there are a number of sharp curve sections, motorists are likely to drive cautiously. To test this theory, a variable for tangent distance between curves was developed.

The Cumulative Tangent Distance between Curves

Finding A decrease in the tangent length before a curve tends to decrease the accident probability for all highways.

Tables 9, 11, 13, 15, 17, 19

As discussed, where drivers have fewer decision points they are less likely to be involved in accidents. Theoretically, motorists are less likely to be involved in an accident on long tangent sections than in sections with many curves. However, the tangent length variable was derived from the assumption that as the tangent length increases before a curve, drivers are less likely to anticipate following curves. Hence, there is an increase in accidents.

<u>The Presence of Curves Greater Than 3 Degrees and Tangent Sections Longer than 0.5 Miles</u>

Finding Sharp curves with more space between them tend to increase the accident probability for minor arterials in Western Washington.

Table 13

This variable tested the interaction of tangent distances and sharp curves by removing sections in which the curve spacing was below 0.5 miles. The findings showed that there is an interaction between tangent length and sharp curves. The relationship is that drivers are more likely to successfully negotiate curves if the curves are spaced closely because they slow down. Conversely, as the space between curves exceeds a certain distance (in this case, 0.5 miles), drivers increase their speeds between the curves and thus create a higher accident probability.

This means that engineers should explore the need for warning signs or other positive guidance tools to help drivers negotiate curves.

<u>The Presence of Curves Greater Than 2 Degrees and Tangent Sections Longer than 0.5 miles.</u>

Finding Curves with more space between them tend to increase the accident probability for principal arterials in Western Washington and collectors in Eastern Washington.

Tables 9, 19

This variable supports the finding of an interaction between tangent length and sharp curves. For this variable, however, interaction was found to be significant for curves greater than 2 degrees rather than 3 degrees.

The Presence of Curves Greater than 10 Degrees and Tangent Sections Longer than 0.25 Miles.

Finding Sharp curves with more space between them tend to increase the accident probability for collectors in Western Washington and decrease the probability for collectors in Eastern Washington.

Tables 17, 19

This indicator variable attempted to capture the combined effects of tangent distances longer than 0.25 miles and curves greater than 10 degrees. The positive coefficient of this variable indicates that as tangent distances increase going into a very sharp curve, accident frequencies are likely to increase. This can be explained by the assumption that when tangents are longer than 0.25 miles before a 10-degree curve, drivers increase their speed to a rate that is at or above the posted speed limit. Thus, travelers have greater difficulty negotiating the sharp curve, which increases the frequency of accidents.

For collectors, which have a greater number of very sharp curves, drivers may anticipate curves and, therefore, take precautions. In either case, it is beneficial to post warning signs or other positive guidance preceding sharp curves.

The Presence of Curves Greater than 3 Degrees and Tangent Sections Longer than 1 Mile.

Finding Sharp curves with more space between them tend to decrease accident probability for minor arterials in Eastern Washington.

Table 15

This variable supports the findings that there is an interaction between tangent lengths and sharp curves. For this variable, interaction was found to be for tangents that were greater than 1 mile.

The Radius of a Curve

Finding A larger curve radius tends to decrease the accident probability for all highways in Washington.

Tables 9, 11, 13, 15, 17, 19

This variable indicates a strong correlation between the radius of a curve and the frequency of accidents. As the radius of a curve increases, the number of accidents decreases. This is intuitively obvious, as a smaller radius creates a sharper curve, which results in greater potential for driver error.

It is apparent from the results that a curve should be designed with the largest radius possible based on location and cost.

__The Central Angle of a Curve__

Finding A decreased central curve angle tends to increase the accident probability for principal arterials in Eastern Washington.

Table 11

The central angle decreases as the curve becomes tighter or sharper. This sharpening exerts additional forces on drivers, making it more difficult for them to maneuver through a curve. The decreased central angle may also reduce the sight distance of a curve. Because of these factors, there is a greater likelihood of accidents.

__The Degree of Curvature__

Finding A higher degree of curvature tends to decrease accident probability for minor arterials in Western Washington.

Table 13

This variable indicates that accidents are less likely on curves with a high degree of curvature. Often, a series of high degree curves are located near each other. Their proximity has the effect of slowing drivers. In addition, these sharp curves are often signed well, warning drivers of an approaching hazard.

__The Length of a Curve__

Finding Longer curves tend to increase the accident probability for collectors and minor arterials in Eastern Washington.

Tables 15, 19

As the length of a curve increases, drivers are required to maintain a proper tracking path for longer periods. Tracking can be affected by drivers misjudging the length of the curve and steering as if the vehicle were in a tangent section. Other factors may be inattention, speeding, or weather conditions.

TABLE 8. DESCRIPTIVE STATISTICS FOR PRINCIPAL ARTERIALS IN WESTERN WASHINGTON MODEL

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Grade1	(grade greater or less than 1%)	-9.3	10.28	0.10966	2.7199
Length	length of a section in feet	0.01	11.41	0.60049E-01	0.13795
Length1	(1 if greater than 0.25 miles, 0 otherwise)	0	1	0.20733E-01	0.14249
AADT	AADT per lane	676	26410	4503.6	4254.7
Medium AADT	(1 if greater than 2500 v., 0 otherwise)	0	1	0.52723	0.49927
Peak Hour	peak hour percentage	6.70	23.80	9.3634	1.6100
Single Trucks	percentage of single unit trucks	1.50	17.40	6.2612	3.5313
Speed	posted speed limit in miles per hour	25	55	49.019	8.0946
Narrow Right Shoulder	(1 if less than ft, 0 otherwise)	0	1	0.33082	0.47634
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.84288	0.36392
Number of Lanes	total number of lanes in a section	1	7	2.6241	0.99481
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.34796	0.47634
Radius	horizontal curve radius in feet	0	50000	1243.4	2674.4
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0	1	0.11934E-01	0.10859
Tangent Length	total length between horizontal curves in miles	0	13.90	0.50310E-01	0.26984

TABLE 9. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR PRINCIPAL ARTERIALS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
Constant		-1.2887 (-8.448)
Grade1	(grade greater or less than 1%)	-0.22304E-01 (-3.918)
Length	length of a section in miles	2.1261 (35.609)
Length1	(1 if greater than 0.25 miles, 0 otherwise)	0.32820 (3.901)
AADT	AADT per lane	0.80791E-04 (21.535)
Medium AADT	(1 if greater than 2500 v., 0 otherwise)	0.23542 (4.474)
Peak Hour	peak hour percentage	-0.60721E-01 (-6.958)
Single Trucks	percentage of single unit trucks	-0.33089E-04 (-4.737)
Speed	posted speed limit in miles per hour	-0.11084E-01 (-5.761)
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.95509E-01 (2.125)
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.25825 (6.132)
Number of Lanes	total number of lanes in a section	0.27056 (14.862)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.42411 (-11.678)
Radius	horizontal curve radius in feet	-0.63380E-04 (-10.491)
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0.71263 (3.080)
Tangent Length	total length between horizontal curves in miles	-2.0754 -15.532
α	dispersion parameter	0.69814 (26.335)
Number of Observations		20248
Restricted Log-Likelihood		-17866.62
Log Likelihood at Convergence		-14160.42

TABLE 10. DESCRIPTIVE STATISTICS FOR PRINCIPAL ARTERIALS IN EASTERN WASHINGTON MODEL

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Length	length of a section in miles	0.01	1.560	0.90466E-01	0.10752
Grade1	(grade greater or less than 2.5%)	-8.00	7.310	0.28344E-01	0.2252
AADT	AADT per lane	252	18200	2244.8	1791.5
Speed	posted speed limit in miles per hour	25	55	52.339	7.0149
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.21900	0.41359
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.86249	0.34440
Number of Lanes	total number of lanes in a section	1	6	2.3993	0.79305
Narrow	(lanes less than 11.5 ft.)	0	1	0.31119	0.46300
Truck	(percentage of all trucks)	1.7	37.03	15.105	7.1672
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.27782	0.44795
Radius	horizontal curve radius in feet	0	40000	1746.8	3104.7
Central Angle	central angle of a curve in degrees	0	137.2	14.716	21.252
Tangent Length	total length between horizontal curves in miles	0	16.37	0.85203E-01	0.43689
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 0.25 mile, 0 otherwise)	0	1	0.67502E-03	0.25974E-01

TABLE 11. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR PRINCIPAL ARTERIALS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
Constant		-3.3882 (-13.287)
Length	length of a section in miles	1.8314 (13.299)
Grade1	(grade greater or less than 2.5%)	0.19036E-01 (2.079)
AADT	AADT per lane	0.14854E-03 (2.079)
Speed	posted speed limit in miles per hour	-0.10295E-01 (-2.642)
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.11683E-01 (3.536)
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.30970 (4.884)
Number of Lanes	total number of lanes in a section	0.12230 (2.115)
Narrow	(lanes less than 11.5 ft.)	0.33692 (12.360)
Truck	(percentage of all trucks)	-0.13789 (-2.559)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.32269 (-4.134)
Radius	horizontal curve radius in feet	-0.67351E-04 (-7.269)
Central Angle	central angle of a curve in degrees	0.25639E-02 (1.715)
Tangent Length	total length between horizontal curves in miles	-2.4655 (-17.293)
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 0.25 mile, 0 otherwise)	2.1394 (5.507)
α	dispersion parameter	0.20909 (6.529)
Number of Observations		11058
Restricted Log-Likelihood		-6824.411
Log Likelihood at Convergence		6266.506

TABLE 12. SUMMARY STATISTICS FOR ACCIDENT FREQUENCY ESTIMATION FOR MINOR ARTERIALS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Length	length of a section in miles	0.01	9.61	0.54277E-01	0.17663
Speed	posted speed limit in miles per hour	25	55	44.387	8.1675
AADT	AADT per lane	187	11020	1536.5	1543.3
Degree of Curve	degree of horizontal curvature	.25	159.2	7.2471	13.330
Tangent Length	total length between horizontal curves in miles	0	33.40	0.46030E-01	0.44732
Tangent Curve	(1 if greater than 3 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0	1	0.63960E-02	0.79724E-01
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.49152	0.49996
Radius	horizontal curve radius in feet	360	22920	846.81	1614.5
Number of Lanes	total number of lanes in a section	1	5	2.0093	0.17266
Narrow Center Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.76529	0.42384

TABLE 13. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR MINOR ARTERIALS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
Constant		-3.4536 (-8.720)
Length	length of a section in miles	0.37113 (8.292)
Speed	posted speed limit in miles per hour	0.92624E-02 (1.958)
AADT	AADT per lane	0.16417E-03 (8.539)
Degree of Curve	degree of horizontal curvature	-0.12478E-01 (-2.593)
Tangent Length	total length between horizontal curves in miles	-3.4017 (-5.052)
Tangent Curve	(1 if greater than 3 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	2.1844 (2.446)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.50838 (-5.168)
Radius	horizontal curve radius in feet	-0.11333E-03 (-4.222)
Number of Lanes	total number of lanes in a section	0.48587 (3.938)
Narrow Center Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.24230 (2.720)
α	dispersion parameter	0.20000 (6.529)
Number of Observations		7192
Restricted Log-Likelihood		-3103.584
Log Likelihood at Convergence		-2955.977

TABLE 14. SUMMARY STATISTICS FOR ACCIDENT FREQUENCY ESTIMATION FOR MINOR ARTERIALS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Length	length of a section in miles	.01	9.9	0.84760E-01	0.19033
Number of Lanes	total number of lanes in a section	2	4	2.0048	0.95002E-01
AADT	AADT per lane	470	4490	604.8	615.67
Speed	posted speed limit in miles per hour	25	55	51.414	7.0573
Single Trucks	percentage of single unit trucks	2.7	21.70	6.6624	3.1472
Peak Hour	peak hour percentage	7.5	20.50	10.918	2.1752
Narrow Lanes	(1 if less than or equal to 11.5 feet, 0 otherwise)	0	1	0.84353	0.36332
Right Shoulder	width of right shoulder in increasing direction in feet	1	38.00	3.1509	2.4227
Tangent Length	total length between horizontal curves in miles	0	12.90	0.63442E-01	0.35591
Radius	horizontal curve radius in feet	0	42980	973.13	1934.2
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.40660	0.49122
Tangent Curve3	(1 if greater than 3 deg. curve and tangent length more than 1 mile, 0 otherwise)	0	1	0.43640E-02	0.65920E-01
Curve Length	length of horizontal curve in feet	54	12470	555.46	4316.1

TABLE 15. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR MINOR ARTERIALS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
CONSTANT		-5.0487 (-6.676)
Length	length of a section in miles	0.41131 (7.926)
Lanes	total number of lanes in a section	0.77954 (3.120)
AADT	AADT per lane	0.59237E-03 (8.948)
Speed	posted speed limit in miles per hour	0.24645E-01 (3.786)
Single Trucks	percentage of single unit trucks	-0.70985E-01 (-3.978)
Peak Hour	peak hour percentage	-0.35703E-01 (-1.630)
Narrow	(1 if less than or equal to 11.5 feet, 0 otherwise)	0.20798 (1.751)
Right Shoulder	width of right shoulder in increasing direction in feet	0.33649E-01 (2.120)
Tangent Length	total length between horizontal curves in miles	-6.6864 (-5.819)
Radius	horizontal curve radius in feet	-0.16978E-03 (-5.106)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.52006 (-5.238)
Tangent Curve3	(1 if greater than 3 deg. curve and tangent length more than 1 mile, 0 otherwise)	8.1820 (2.658)
Curve Length	length of horizontal curve in feet	0.10518E-04 (2.294)
α	dispersion parameter	0.20909 (6.529)
Number of Observations		9919
Restricted Log-Likelihood		-2460.011
Log Likelihood at Convergence		-2264.545

TABLE 16. SUMMARY STATISTICS FOR ACCIDENT FREQUENCY ESTIMATION FOR COLLECTORS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Length	length of a section in miles	0.01	11.30	0.58742E-01	0.14245
AADT	AADT per lane	301	19640	3053.2	2686.9
Peak Hour	peak hour percentage	6.310	15.20	9.8985	1.6795
Train Truck	percentage of tractor plus semitrailer plus full trailer trucks	0.02	12.05	0.46099	0.46154
Double Truck	percentage of tractor plus one semitrailer trucks	0.42	12.00	2.5176	1.6072
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.98904	0.10410
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0	1	0.41833	0.49331
Number of Lanes	total number of lanes in a section	1	7	2.1122	0.50574
Narrow lanes	(1 if less than or equal to 11.5 feet, 0 otherwise)	0	1	0.62928	0.48302
Tangent Length	total length between horizontal curves in miles	0	10.30	0.49254E-01	0.26768
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 1 mile, 0 otherwise)	0	1	0.33865E-02	0.58098E-01
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.35777	0.47937
Radius	horizontal curve radius in feet	72	28650	951.06	1921.0

TABLE 17. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR COLLECTORS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
Constant		-2.1532 (-6.666)
Length	length of a section in miles	.35108 (6.987)
AADT	AADT per lane	0.11533E-03 (12.745)
Peak Hour	peak hour percentage	-0.14191 (-7.454)
Train Truck	percentage of tractor plus semitrailer plus full trailer trucks	-0.23902 (-3.360)
Double Truck	percentage of tractor plus one semitrailer trucks	0.56510E-01 (2.640)
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.85406 (4.335)
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.10940 (2.130)
Number of Lanes	total number of lanes in a section	0.38603 (11.831)
Narrow lanes	(1 if less than or equal to 11.5 feet, 0 otherwise)	0.17291 3.239
Tangent Length	total length between horizontal curves in miles	-0.96803 (-10.924)
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 1 mile, 0 otherwise)	-1.9381 (-2.874)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.47885 (-8.081)
Radius	horizontal curve radius in feet	-0.10962E-03 (-6.708)
α	dispersion parameter	0.57841 (11.143)
Number of Observations		10158
Restricted Log-Likelihood		-6298.680
Log Likelihood at Convergence		-5652.977

TABLE 18. SUMMARY STATISTICS FOR ACCIDENT FREQUENCY ESTIMATION FOR COLLECTORS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		MIN.	MAX.	MEAN	STD. DEV.
Length	length of a section in miles	0.01	8.98	0.95565E-01	0.13969
Median	(1 if divided, 0 otherwise)	0.0000	1.000	0.31751	0.46553
AADT	AADT per lane	90	10440	958.17	965.49
Speed	posted speed limit in miles per hour	25	55	52.516	6.4147
Double Trucks	percentage of tractor plus one semitrailer trucks	0.18	20.64	5.5309	4.2134
Train Trucks	percentage of tractor plus semitrailer plus full trailer trucks	0.02	9.460	1.3828	1.9680
Number of Lanes	total number of lanes in a section	1	4	2.0525	0.32038
Lane Width	width of a lane in feet	9.0	35.00	11.664	1.9458
Wall	(1 if wall, 0 otherwise)	0	1	0.36752E-01	0.18816
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	0	1	0.39202	0.48823
Radius	horizontal curve radius in feet	0	50000	1547.1	3575.8
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0	1	0.17212E-01	0.13007
Tangent Length	total length between horizontal curves in miles	0	14.30	0.77903E-01	0.36478
Curve Length	length of a curve in feet	520	6949	468.24	668.98

TABLE 19. NEGATIVE BINOMIAL ACCIDENT FREQUENCY ESTIMATION FOR COLLECTORS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		COEFFICIENT (T-STAT)
Constant		-3.7406 (-7.583)
Length	length of a section in miles	0.25379 (1.810)
Median	(1 if divided, 0 otherwise)	-0.16967 (-2.166)
AADT	AADT per lane	0.22999E-03 (7.543)
Speed	posted speed limit in miles per hour	0.10886E-01 (-1.520)
Double Trucks	percentage of tractor plus one semitrailer trucks	-0.35008E-01 (-2.490)
Train Trucks	percentage of tractor plus semitrailer plus full trailer trucks	0.59886E-01 (2.364)
Number of Lanes	total number of lanes in a section	0.29231 (4.143)
Lane Width	width of a lane in feet	0.47787E-01 (2.248)
Wall	(1 if wall, 0 otherwise)	-0.51188 (-1.870)
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.84176 (-8.859)
Radius	horizontal curve radius in feet	-0.16272E-03 (-10.847)
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	2.3690 (3.282)
Tangent Length	total length between horizontal curves in miles	-5.0552 (-13.407)
Curve Length	length of a curve in feet	0.25492E-03 (3.971)
α	dispersion parameter	0.20000 (5.081)
Number of observations		9416
Restricted Log-Likelihood		-3333.783
Log Likelihood at Convergence		-3117.429

Using the regression coefficients, the predicted accidents were compared with actual accidents. The models developed show a reasonable ability to predict accidents on the basis of the geometric and traffic flow characteristics of the highway. Table 20 shows the comparison.

TABLE 20. PREDICTED VERSUS OBSERVED ACCIDENTS

Highway	Predicted	Observed
W. Wash. Principal Arterials	8278	8979
E. Wash. Principal Arterials	2253	2778
W. Wash. Minor Arterials	1111	1004
E. Wash. Minor Arterials	662	662
W. Wash. Collectors	2526	2621
E. Wash. Collectors	993	1013

ELASTICITIES

The concept of elasticity provides the modeler with the ability to determine the effect a 1 percent change in an independent variable will have on accident frequency. The classic definition of direct elasticity is given by the equation

$$E_{x_{ij}}^{\lambda_i} = \frac{\partial \lambda}{\partial x} \bullet \frac{x_{ij}}{\lambda_i} \qquad \text{(eq. 9)}$$

such that λ_i is the mean accident frequency at section i and is determined by equation 4, where $\lambda_i = \exp(\beta X_i + \varepsilon_i)$, and x_{ij} is the value of the geometric or roadway variable j.

Differentiating equation 4 with respect to equation 9 gives the following:

$$E_{x_{ij}}^{\lambda_i} = \beta_j x_{ij} \qquad \text{(eq. 10)}$$

Similarly, analysts may want to determine the elasticity of an indicator variable. The "pseudo elasticity" of the indicator variable is defined intuitively from $1 \rightarrow 0$, whereas the mathematical derivation is actually from $0 \rightarrow 1$. The pseudo elasticity gives us the incremental change in accident frequency caused by the presence of a condition described by the indicator variable. The pseudo elasticity equation is given by

$$E_{x_{ij}}^{\lambda_i} = \frac{\exp(\beta) - 1}{\exp(\beta)} \qquad \text{(eq. 11)}$$

The elasticities for each independent variable were determined for the differing functional class models and are shown in tables 21 through 26.

Principal Arterials

The principal arterial model for Western Washington consisted of 15 independent variables, of which six were indicator variables and nine were continuous explanatory variables. Average elasticities for all of the variables are presented in Table 21.

The results of the model indicated that no variable was elastic. That is, no variable had an absolute value that was greater than 1.0. This does not imply that improvements should not be undertaken. The elasticities suggest the relative importance each variable has on increasing or decreasing the accident frequency.

Three elasticities were above 0.5. The variables were peak hour, number of lanes, and speed. It is important to realize that each of these variables has a direct relationship to traffic flow. For example, an increase in speed will decrease the number of accidents. The interpretation of this variable is not that the faster drivers go, the safer they are. Rather, many lower-speed facilities have significantly more access points and intersection controls that may affect the facility adversely. Fewer access points and intersections reduce conflicts and accidents. This allows for higher safe speed limits.

The variable for the number of lanes produced an elasticity number of 0.71, which indicates that a 1 percent change in the number of lanes will be accompanied by a 0.71 percent change in the number of accidents.

Like the principal arterials in Western Washington, those in Eastern Washington did not produce any variables with an elasticity of greater than 1.0. However, two variables had elasticities of above 0.5.

Once again, the variable for the number of lanes showed a relatively strong tendency to increase accidents when the number of lanes increased, with an elasticity of

0.81. Similarly, as speed increased, accidents tended to decrease, and the variable had an elasticity of 0.61. The strongest of the indicator variables in this model was Narrow Left Shoulder of a two-lane highway, with an elasticity of 0.23.

Minor Arterials

Unlike principal arterials, the number of lanes variable for Eastern Washington was elastic, with a value of 1.56, and was nearly elastic in Western Washington, with a value of 0.97. These values indicate that a 1 percent increase in the variable will produce a 1.56 and 0.97 percent increase in the frequencies of accidents, respectively. For Eastern Washington, the speed variable was also elastic, with an absolute value of 1.26. This indicates that a 1 percent increase in the speed limit will reduce accident frequency by 1.26 percent.

Collectors

For collector highways in Western Washington, the peak hour variable was elastic. This means that a 1 percent increase in peak hour corresponds to a 1.41 percent decrease in the frequency of accidents. Other variables within these two models were not found to be elastic. The most significant of the variables was number of lanes, with elasticities of 0.82 and 0.60 for Western and Eastern Washington, respectively.

TABLE 21. ELASTICITIES ESTIMATION FOR PRINCIPAL ARTERIALS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Grade1	(grade greater or less than 1%)	-0.24458E-02
Length	length of a section in miles	0.12767
Length1	(1 if greater than 0.25 miles, 0 otherwise)	0.18260E-01
AADT	AADT per lane	0.36385
Medium AADT	(1 if greater than 2500 v., 0 otherwise)	0.11059
Peak Hour	peak hour percentage	-0.56855
Single Trucks	percentage of single unit trucks	-0.20718
Speed	posted speed limit in miles per hour	-0.54333
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.30134E-01
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.19184
Number of Lanes	total number of lanes in a section	0.70998
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.18381
Radius	horizontal curve radius in feet	-0.78805E-01
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0.60823E-02
Tangent Length	total length between horizontal curves in miles	-0.10441

TABLE 22. ELASTICITIES ESTIMATION FOR PRINCIPAL ARTERIALS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Length	length of a section in miles	0.16568
Grade1	(grade greater or less than 2.5%)	0.53956E-03
AADT	AADT per lane	0.33344
Speed	posted speed limit in miles per hour	-0.15550
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	-0.61148
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.22971
Number of Lanes	total number of lanes in a section	0.25210E-01
Narrow	(lanes less than 11.5 ft.)	0.80838
Truck	(percentage of all trucks)	-0.42909E-01
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.10732
Radius	horizontal curve radius in feet	-0.11766
Central Angle	central angle of a curve in degrees	0.37731E-01
Tangent Length	total length between horizontal curves in miles	-0.21007
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 0.25 mile, 0 otherwise)	0.59556E-03

TABLE 23. ELASTICITIES ESTIMATION FOR MINOR ARTERIALS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Length	length of a section in miles	0.20144E-01
Speed	posted speed limit in miles per hour	0.41113
AADT	AADT per lane	0.25225
Degree of Curve	degree of horizontal curvature	-0.90430E-01
Tangent Length	total length between horizontal curves in miles	-0.15658
Tangent Curve	(1 if greater than 3 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0.56762E-02
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.11782
Radius	horizontal curve radius in feet	-0.95969E-01
Number of Lanes	total number of lanes in a section	0.97627
Narrow Center Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.16468

TABLE 24. ELASTICITIES ESTIMATION FOR MINOR ARTERIALS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Length	length of a section in miles	0.32894E-01
Number of Lanes	total number of lanes in a section	1.5619
AADT	AADT per lane	0.34422
Speed	posted speed limit in miles per hour	-1.2679
Single Trucks	percentage of single unit trucks	-0.47370
Peak Hour	peak hour percentage	-0.39105
Narrow	(1 if less than or equal to 11.5 feet, 0 otherwise)	0.18092
Right Shoulder	width of right shoulder in increasing direction in feet	0.10439
Tangent Length	total length between horizontal curves in miles	-0.45092
Radius	horizontal curve radius in feet	-0.17077
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.28749
Tangent Curve3	(1 if greater than 3 deg. curve and tangent length more than 1 mile, 0 otherwise)	0.45602E-02
Curve Length	length of horizontal curve in feet	0.55534E-02

TABLE 25. ELASTICITIES ESTIMATION FOR COLLECTORS IN WESTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Length	length of a section in miles	0.20623E-01
AADT	AADT per lane	0.35212
Peak Hour	peak-hour percentage	-1.4047
Train Truck	percentage of tractor plus semitrailer plus full trailer trucks	-0.11019
Double Truck	percentage of tractor plus one semitrailer trucks	0.14227
Narrow Left Shoulder	(1 if less than 5 ft, 0 otherwise)	0.56802
Narrow Right Shoulder	(1 if less than 5 ft, 0 otherwise)	0.43350E-01
Number of Lanes	total number of lanes in a section	0.81535
Narrow lanes	(1 if less than or equal to 11.5 feet, 0 otherwise)	0.10881
Tangent Length	total length between horizontal curves in miles	0.47679E-01
Tangent Curve3	(1 if greater than 10 deg. curve and tangent length more than 1 mile, 0 otherwise)	-0.20134E-01
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.21975
Radius	horizontal curve radius in feet	-0.10426

TABLE 26. ELASTICITIES ESTIMATION FOR COLLECTORS IN EASTERN WASHINGTON

INDEPENDENT VARIABLE		ELASTICITIES
Length	length of a section in miles	0.24254E-01
Median	(1 if divided, 0 otherwise)	-0.53871E-01
AADT	AADT per lane	0.22037
Speed	posted speed limit in miles per hour	0.57169
Double Trucks	percentage of tractor plus one semitrailer trucks	-0.19363
Train Trucks	percentage of tractor plus semitrailer plus full trailer trucks	0.82809E-01
Number of Lanes	total number of lanes in a section	0.59998
Lane Width	width of a lane in feet	0.55737
Wall	(1 if wall, 0 otherwise)	-0.18813E-01
Sharp Curve	(1 if greater than 2 deg. curve, 0 otherwise)	-0.51764
Radius	horizontal curve radius in feet	-0.25174
Tangent Curve2	(1 if greater than 2 deg. curve and tangent length more than 0.5 mile, 0 otherwise)	0.15601E-01
Tangent Length	total length between horizontal curves in miles	-0.39382
Curve Length	length of a curve in feet	0.11936

CHAPTER 5
SUMMARY AND RECOMMENDATIONS

SUMMARY

The intent of the research was to provide an overview of the effects of highway geometrics on safety. The results will soon form the basis for the new Risk sub-program for use in prioritizing and programming highway safety projects throughout Washington State.

In researching the effects of highway geometrics on safety, the researchers performed an extensive literature review to determine the acceptability of different statistical methods and possible variables for use in the models. The literature review indicated a need to change from the use of linear regression to the Poisson and negative binomial regression methods. The literature suggested that the Poisson regression and the negative binomial models possess most of the desirable statistical properties in describing vehicle accident events. However, one of the stated limitations of the Poisson regression model is that the variance of the accident data is constrained to equal the mean. As a result, the variances of the estimated model coefficients for count data are often over- or underdispersed relative to the mean.

This was the case with results from the models developed during this research, which showed that each Poisson regression model was overdispersed relative to the mean. Therefore, as the literature review suggested, the negative binomial regression was chosen for modeling the interaction between geometrics, travel data, and accidents.

The researcher developed the negative binomial models using data from 80,795 highway sections and over 80,000 accidents. Each highway section contained 45 independent variables. From these data, an additional 16 indicator variables were identified, and variable interactions were tested.

Each model was developed with the belief that by identifying accident locations and comparing them to the roadway characteristics, locations likely to have high accident rates could be determined. WSDOT could then program safety money before a substantial accident history developed.

As stated, the negative binomial regression provides an acceptable means of predicting accident frequencies. The estimated negative binomial models showed that the relationship between vehicle crashes and highway geometric and traffic flow variables include the following key variables: the number of lanes, horizontal curvature, speed, tangent length, section length, AADT, and peak hour.

RECOMMENDATIONS

WSDOT intends to spend each safety dollar as effectively and efficiently as possible. To achieve this result, the Department must program and spend its safety dollars at locations that provide the greatest benefit. This effort will require continuous tracking of safety investments. Research should evaluate trends in Washington's accident rates to determine whether improvement has actually occurred.

The WSDOT Olympia Service Center (OSC) currently uses the risk model. The OSC predicts the accident frequency for every highway in the state. Once the accident frequencies have been predicted, each highway section is ranked from most accidents to least. The highways sections that are above the critical rate (average) for similar highways in the state are described on a ranked list. This list is sent to the WSDOT regional offices so that they can perform a roadside clear zone review and analysis. Using this information from the regions, the probable accident severity is determined for the section. The accident severity is in the form of societal cost per accident. Multiplying the probable accident severity by the predicted frequency produces an associated societal cost for the highway section.

Even though WSDOT requires associated accident severities to be included during the prioritization process, it does not maintain a database for this information.

Logically then, WSDOT should develop a means to collect and rate roadside information. It is suggested that a research project be developed to review state of the art practices for collecting roadside information and to develop a format for collecting and maintaining the information. Following this, additional research should be completed to rework the risk models so that they include accident severity. These two research projects will drastically reduce the resources WSDOT expends collecting and producing programming information each biennium.

Finally, the research completed in this project, and the recommendations for additional research, will ensure WSDOT that each safety dollar spent achieves the greatest benefit possible.

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